Structural analysis of earth architecture

A report on the structural performance of structures designed from adobe bricks for Al'Zaatari refugee camp

Faculty of architecture and built environment
Technical University of Delft

Course code: AR3B011 EARTHY

1. Introduction

The syrian civil war which erupted in March 2011 has created one of the largest humanitarian problem of our time. More than 6.6 million syrians, have been forced to flee from their homes (Mercy Corps, 2018) causing them to be the largest refugee population in the world. While these refugees have been forced to scattered all over the world, with most of the refugees finding their way to camps in Turkey, Lebanon, and Jordan. One of these camps is the Zaatari camp, located in the north of Jordan. This camp was opened to the refugees on 28 July 2012. With an initial population of 15000 in 2012, Zaatari now hosts 76414 people (UNHCR,2019). During its 7 years of existence, the camp is slowly changing from a temporary camp to a more permanent settlement.

This change was driven by the wish of the inhabitants to have better living conditions, this started when the tents were exchanged for container houses. The residents then started to further improve their homes sometimes through illegally dismantling the public washing facilities. To guide the further development of the camp a better method has to be found to enable the residents wish while keeping in mind the limited resources in the camp, by making the most of the materials available.

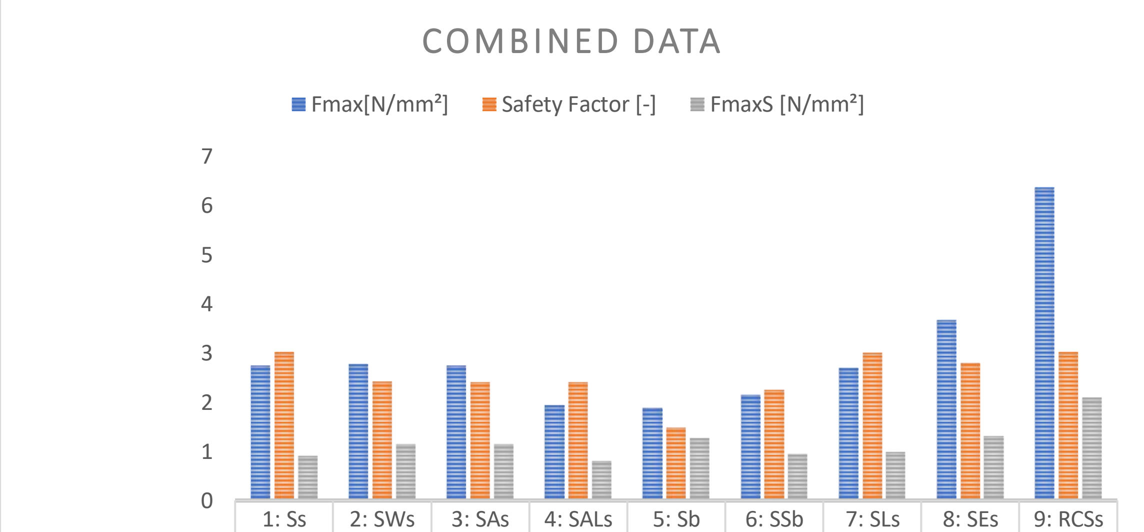
The aim of this report is to structurally validate the adobe structure designed for the Zaatari camp during the master course earthy.

2. *Structural* Analysis

After finalising the shape through dynamic relaxation in grasshopper the geometry was put into Karamba a finite element method(FEM) plugin that enabled us to test the geometry for internal stresses. The most crucial result of the FEM was the internal tensile stress, as that is the weakest property of adobe. In Karamba it is important to look at the first and second principle stresses directions revered to ass stress 1 and stress 2

The tested parts of the building are separated into three categories: stoa, domes, and vaults. During the construction all these categories need to be able to be constructed structurally independent form the others, the stoa is an exception as it will be constructed last it will always have the either domes or vault to counteract the horizontal loads while the domes and vaults need to be able to stand on their own.

Because most of us had no prior experience with adobe a brick making and a brick breaking workshop were implemented. In the table below you can see the results of the bricks made by our group. For a more in-depth explanation of the workshops and the results see our material testing report.



	1: Ss	2: SWs	3: SAs	4: SALs	5: Sb	6: SSb	7: SLs	8: SEs	9: RCSs
Fmax[N/mm²]	2,75	2,77	2,74	1,94	1,88	2,15	2,69	3,67	6,37
Safety Factor [-]	3,02	2,42	2,4	2,41	1,48	2,25	3,01	2,8	3,02
FmaxS [N/mm²]	0,91	1,14	1,14	0,8	1,27	0,95	0,98	1,31	2,1

Form the samples tested we decided to use the values from the large bricks with the standard mixture.

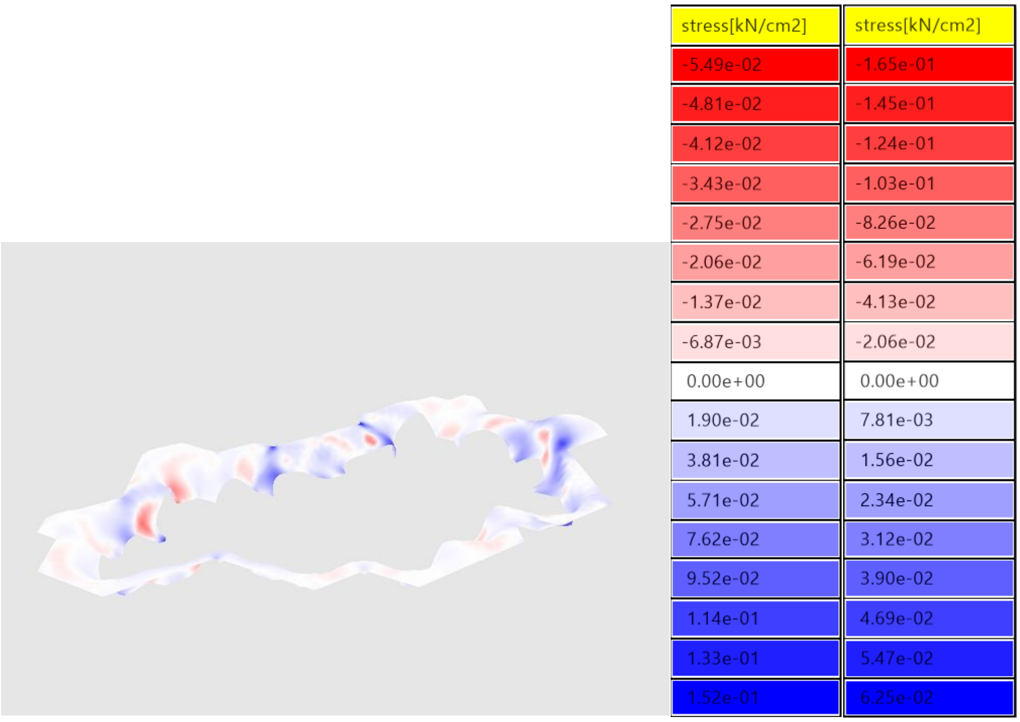
Resulting in the following starting values.

Youngs modulus	7,6	[Mpa]
Density	1520	[kg/m3]
maximum compressive strenght	1,88	[N/mm2]
compressive strengh after safety factor	1,27	[N/mm2]

For the structural analysis we assumed that our masonry structure would function as a shell made out of Adobe.

Stoa

Using these value's and a thickness om 20cm for our 'shell' we got the following results for the structural analysis of the Stoa, our semi open space.



After running the first analysis with the values in table 2 we concluded that our young's modulus was incredibly low. According to literature the maximum allowable tensile stress for earth constructions is between 10%(Martins, T.,2006) and 20%(NZ 4297,1998) of the compressive strenght. Meaning that the maximum tensile stress should be below one fifth of the compressive strength.

maximum tensile stress	0,254	[N/mm2]

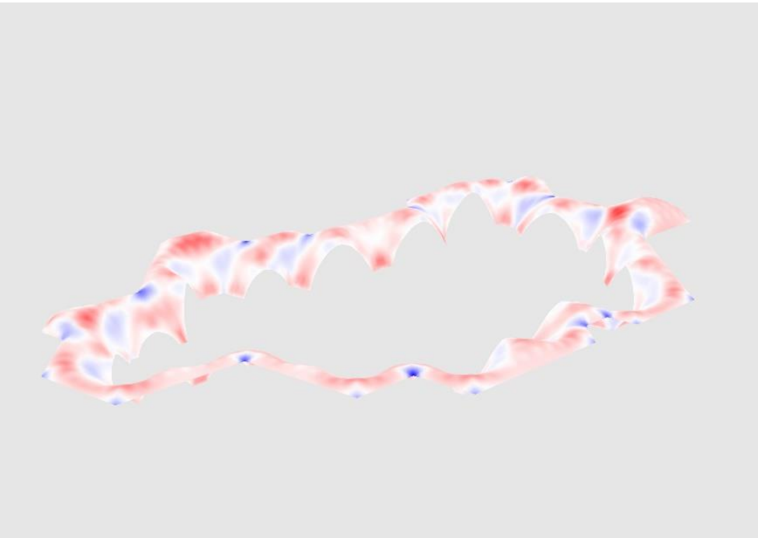
Looking at the results from the analysis in table 4 we can say that we not only exceed the maximum tensile stress but also exceed the maximum compressive strength with the applied safety factor

Found stresses					
maximum compressive stress 1	0,54	[MPa]	maximum compressive stress 2	1,65	[MPa]
maximum tensile stress 1	1,52	[MPa]	maximum tensile stress 2	0,63	[Mpa]

Following these results we went back into the literature and found that our Young's modulus was incredibly low, according to multiple tests that have been done all over the world the property of adobe that was used in construction can be anywhere in between 50 and 250 MPa.

Taking the literature into account we decided to increase our Young's modulus by ten times to 76MPa. This is well within the margin that was found through the literature research.

With this Young's modulus we tried again and got the following results



stress[kN/cm2]	stress[kN/cm2]
-8.39e-03	-4.79e-02
-7.34e-03	-4.19e-02
-6.29e-03	-3.59e-02
-5.24e-03	-3.00e-02
-4.19e-03	-2.40e-02
-3.15e-03	-1.80e-02
-2.10e-03	-1.20e-02
-1.05e-03	-5.99e-03
0.00e+00	0.00e+00
1.48e-03	2.99e-04
2.95e-03	5.98e-04
4.43e-03	8.97e-04
5.91e-03	1.20e-03
7.38e-03	1.50e-03
8.86e-03	1.79e-03
1.03e-02	2.09e-03
1.18e-02	2.39e-03

These results where now within the acceptable range but the analysis still found tensile stresses in places where the structure needed to be compression only, therefore further tests where done with other thicknesses of the cross-section looking at the maximum stresses and deformation. The deformation was important because a material like adobe deforms over time thus making it important to take into account the starting deformation, this is mostly important for buildings with a second floor and for buildings with complex roofs such as this project.

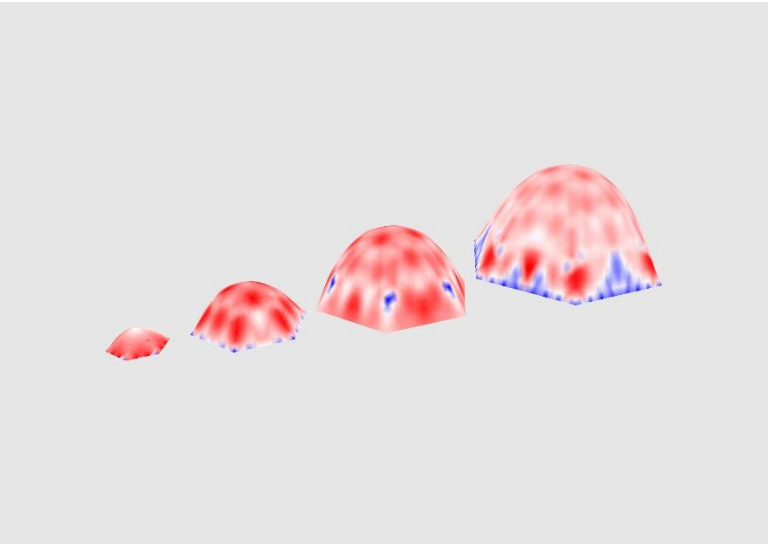
Taking the construction into account the tested thicknesses were 20cm, 30cm, and 40cm. While running these tests it became apparent that the cross-section thickness did not have a linear relation to the tensile and compressive stresses. In an attempt to even out the differences the cross-section was optimised for material use and deformation. All these tests gave the following maximum results.

Thickness	compressive stress 1 [MPa]	compressive stress 2 [MPa]	tensile stress 1 [MPa]	tensile stress 2 [MPa]	defomation [cm]
20cm	0,083	0,479	0,118	0,024	1,75
30cm	0,099	0,394	0,114	0,022	1,43
40cm	0,108	0,354	0,116	0,023	1,24
optimised	0,078	0,163	0,104	0,021	0,76

By optimizing the stoas cross-section one makes it structurally more stable but it also becomes harder to build, with a thickness ranging between 20cm and 80cm the masonry becomes a challenge of it's own. Seeing as this is a optimised computational model it will be safe to say that in reality the amount changes in thickness will be less fluent than the calculated model.

Domes

In the construction principles of the project it was decided that all the rooms and domes should be capable of standing independently. In the project there are four different dome spans: 1.6m, 3.6m, 6.2m, and 8.6m label from smallest to largest Dome 1 to 4



For the domes the analysis was done again to see the influence of thickness of the cross-section.

A strange thing that happened in the analysis was that no tensile stress was found in the second principle direction. This is most definitely an error in the model, But even after looking through it for multiple days no answers were found on how to solve it.

10cm thickness	compressive stress 1 [MPa]	tensile stress 1 [MPa]
Dome 1	0,031	0,008
Dome 2	0,050	0,012
Dome 3	0,077	0,031
Dome 4	0,167	0,082
20cm thickness	compressive stress 1 [MPa]	tensile stress 1 [MPa]
Dome 1	0,021	0,010
Dome 2	0,034	0,012
Dome 3	0,044	0,024
Dome 4	0,057	0,031
30cm thickness	compressive stress 1 [MPa]	tensile stress 1 [MPa]
Dome 1	0,018	0,011
Dome 2	0,029	0,012
Dome 3	0,038	0,022
Dome 4	0,044	0,027

Based on these results all domes suffice but the values are really low indicating that there might still be errors in the Karamba model. Further research has to be done to prove the validity of these results. But to allow the project to continue a thickness of 20cm was chosen as it allows for a bit more inaccuracy in the masonry taking the unskilled laborers in account.

For the constructability of the domes it is important that the forces are driven down through the walls properly, if this is not the case then either the wall has to become thicker, the dome has to become higher or mass needs to be added at the edges of the dome to shift the forces more downwards.

The rule of thumb used to specify the wall thickness was taken from literature on traditional Arabic architecture(Memarian, G. 1988) it stated that the walls needed to have the thickness of one sixth of the span of the dome. Giving us four different wall thicknesses, from smallest to largest gave us the following wall thicknesses: 50cm, 80cm, 100cm, and 120cm.

For this calculation it was important to know that the height from dome to ground is 2.2m and the reactionary forces that the domes deliver to the walls. From Karamba we get the reaction forces in the x and z axis. For this calculation however we only care about the 2D representation of these forces the horizontal and vertical reactionary force. The forces as seen in the table below were chosen because they were the pairs with the smallest difference between horizontal and vertical component.

Reaction forces	dome 1	dome 2	dome 3	dome4
horizontal	1,79	2,87	5,84	7,56
vertical	2,72	6,01	12,82	17,58

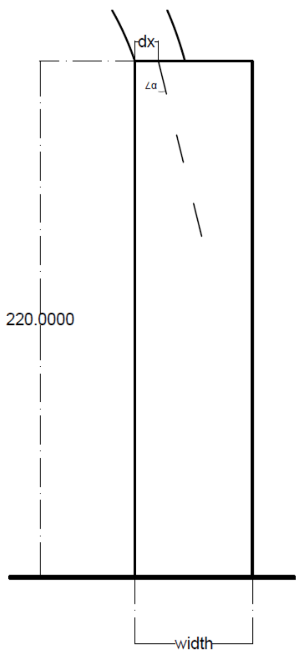
Using Pythagoras theorem we can identify the angle of incidence of the true reactionary force of the dome and see if the wall is thick enough of if actions have to be taken.

We no that the angel of incidence is the tangent of the horizonal load over the vertical load.

$$\angle\alpha = tan^{-1}\left(\frac{horizontal\ reactionry\ force}{vertical\ reactionry\ force}\right)$$

Following this we can say that the required thickness of the wall should be equal to the result of tan∠α multiplied with the height of the wall plus δx. With δx being the half the shell thickness, as the forces run trough the center of the profile

$$width\ wall = tan\angle\alpha \times height\ of\ the\ wall + \delta x$$



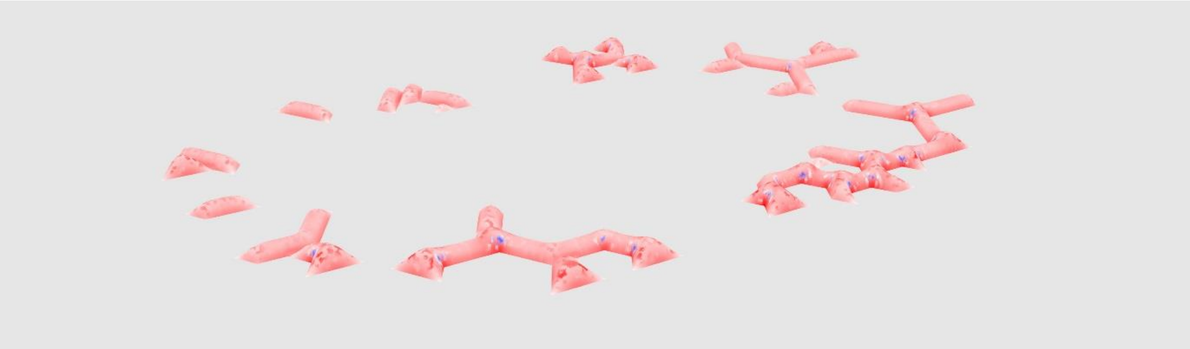
following these steps we get the resulting wall thicknesses

wall thickness cm	dome 1	dome 2	dome 3	dome 4
rule of tumb	50	80	100	120
calculated	154	115	110	104

Following these results we can say that the rule of thumb only works with domes of sufficient heights. Looking at the project it is safe to say that a lot of mass would have to be added for the domes to function on their own in their current geometry.

Vaults

For the vaults a similar approach was used as with the domes, and a similar situation occurred in the vaults with only the primary principle stress showing tensile stress and on a very low level. With the peak tensile stresses being in the intersections between the straight vaults.



Thickness	compressive stress [MPa]	tensile stress [MPa]	defomation [cm]
10cm	0,015	0,006	0,07
20cm	0,011	0,004	0,04
30cm	0,011	0,004	0,04

During the design we were assigned the task to redesign the point where the vaults intersect through muqarnas domes. This would make the intersections their own geometry leaving us only with straight vaults.

Conclusion

The first analysis can be said to have been a proof of concept, with such a low young's modulus no building would be able to stand. After increasing the young's modulus the calculations showed potential. But with the analysis of both the domes and the vaults being a point of debate over their validity due to the results being unrealistic more importance has to be placed on the analysis of the stoa. The stoa had already been the main focus for pushing what was possible in adobe, with is asymmetric geometry and non-uniform span. Through optimizing the thickness the unwanted stresses were prevented in placed where the structure functions under compression only.

Realistically with two of the three main calculations ending up as being invalid it hard to said with certainty that the stoa calculation can be trusted. What can be said is that the tensile stresses that appeared in the stoa seemed to logical.

Reflection

Further investigation has to be done into the structural integrity of the domes and vaults. After trouble shooting for two weeks it appears to be related with the tessellation and the corresponding anchor point. most likely the idea of having the entire edge of the mesh function as loadbearing is not something that works properly in Karamba. But even after removing a over half the anchor points from the calculation the values were still unrealistic. Maybe another FEM program like Ansys could have helped us figure out if the results were accurate.

Another part of the project that was not addressed was the added load due to the second layer on top of the structure(roof) because the thickness of the roof was non uniform and the geometry was only finalised at the end of the project the time was not there to add it to the calculations of the structure.

An attempt was made to calculate all categories together but due to differences in the mesh tessellation this was impossible as the load could not properly flow from one mesh to the other when vertices misalign. This is something that would have to be prevented during the forming part of the project.

References

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